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SPUD POINT MARINA BREAKWATER, BODEGA BAY SONOMA COUNTY, CALIFORNIA

by

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13. ABSTRACT *(Maximum 200 words)*

A field monitoring study of the Spud Point Marina breakwater was conducted as part of the Monitoring Completed Coastal Projects Program. The breakwater is located within the confines of Bodega Harbor, a relatively protected embayment on the California coast north of San Francisco. The concrete pile-supported structure was selected for monitoring because of its unusual baffled design. Openings in the breakwater below the mean lower low tide level permit relatively unimpeded marina flushing. The baffle panel submergence depth was chosen using theoretical wave height transmission results. A field study of wave transmission was conducted using boat wakes and pressure sensors to measure the generated waves. Soundings of potential scour zones and a side-scan sonar survey were made. Circulation through the breakwater and marina was measured, and the breakwater was examined for structural integrity. Unexpectedly high dissipation of generated waves as they crossed a shallow region fronting the breakwater prevented quantification of wave transmission performance. Flushing performance appeared to be satisfactory. No evidence of scour or

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structural displacement was found; however, hairline cracks were observed in the cap, which should be monitored as part of future Operations and Maintenance. From this study and other evidence, it appears that the breakwater is giving satisfactory wave attenuation performance. Designers of breakwaters in low-energy environments where flushing performance is critical should consider a baffled structure, although physical model testing may be needed to more confidently predict wind wave and boat wake attenuation. The design approach cannot be deemed more generally applicable, since the contribution of site-specific conditions (particularly the shallow flats fronting the breakwater) to good wave attenuation performance is unknown.

Contents

Figures

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
degrees (angle)	0.1745329	radians
feet	0.3048	metres
miles	1.609347	kilometres

Preface

This report was prepared as part of the Monitoring Completed Coastal Projects (MCCP) Program. The program calls for intensive monitoring of selected Civil Works coastal projects, to assure adequate information as a basis for improving project purpose attainment, design procedures, construction methods, and operations and maintenance techniques. Overall program management is by the Hydraulic Design Section of Headquarters, US Army Corps of Engineers (HQUSACE). The Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station (WES), is responsible for technical and data management control and support for HQUSACE, review and technology transfer. The Spud Point Marina Breakwater Project is located within the jurisdiction of the US Army Engineer District, San Francisco (SPN). The work was carried out under the Spud Point Marina breakwater monitoring effort (also referred to as Bodega Bay), Work Unit 22123. For the MCCP Program, the HQUSACE Technical Monitors are Messrs. John H. Lockhart, Jr.; John G. Housley; James E. Crews; and Robert H. Campbell. The MCCP Program Manager is Ms. Carolyn M. Holmes, succeeding Mr. J. Michael Hemsley of CERC.

The report presents the results of the MCCP effort focusing on the unusual concrete pile breakwater fronting Sonoma County, California's Spud Point Marina facilities. The breakwater was built to provide wave protection for the marina located inside a bay (Bodega Harbor). The design accomplishes wave protection while allowing flushing, through the use of baffled openings between the supporting piles.

This MCCP effort was performed under the general direction of Dr. James R. Houston, Chief, CERC, and Mr. Charles C. Calhoun, Jr., Assistant Chief, CERC, and under the direct supervision of Mr. Thomas W. Richardson, Chief, Engineering Development Division (EDD), CERC, and Mr. William L. Preslan, Chief, Prototype Measurement and Analysis Branch (PMAB), EDD, CERC. The Principal Investigator (PI) was originally Mr. Hemsley. Mr. William S. Hegge took over as PI, followed by Mr. Jonathan W. Lott, PMAB. The San Francisco District partner in this effort was Mr. Thomas Kendall. Valuable contributions were made by Mr. Richard Lou and Ms. Emy Tatami of SPN. The field experiments for the MCCP study were a team effort between CERC's PMAB, SPN's Construction and Operations Division, and the US Coast Guard station at Doran Point, California.

This report was prepared by Mr. Lott based on material assembled by the previous PIs and Mr. Kendall. Moffatt & Nichol, Engineers, Long Beach, CA, provided copies of wave transmission calculations to Mr. Kendall.

COL Larry B. Fulton, EN, is Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

1 Introduction

Project Location and Description

Spud Point Marina breakwater is located in the northwestern part of Bodega Harbor, an enclosed bay on the California coast, about 60 miles[1] north of San Francisco (Figure 1). Bodega Harbor serves light-draft vessels and is the only harbor of refuge in the 150-mile stretch of coastline between San Francisco Bay and Noyo Harbor. Bodega Harbor is protected from the open Pacific Ocean by a rocky headland and peninsula, Bodega Head. A sandy peninsula across the southern part of Bodega Harbor, Doran Beach, separates it from the area of the open coast known as Bodega Bay. Access to the various marina facilities inside Bodega Harbor is provided through a jettied entrance and dredged navigational channels. Bodega Harbor is relatively shallow; the deepest parts are the dredged channels and turning basins. These channels and basins have a nominal project depth of -12 ft mean lower low water (mllw). Figure 2 is an oblique aerial photo from above Bodega Head looking northeast taken February 8, 1987. The large extent of tidal flats, the channels, entrance, and Doran Point US Coast Guard (USCG) Station (center right of photo) are visible. Spud Point Marina and breakwater are at the upper left. Some of the other marinas and the town of Bodega Bay on the northeast side of the harbor are also shown. The marina docks and shoreside facilities are operated by Sonoma County. The breakwater and access channel are maintained by the Corps of Engineers in addition to the entrance jetties and other dredged channels and basins shown in Figure 1.

The breakwater is a concrete pile baffled structure consisting of prestressed concrete vertical piles and a cast-in-place concrete cap beam, with prestressed concrete baffle panels between the vertical support piles under the cap beam. The breakwater cap beam has handrails along its entire length and wooden platforms near the northern end and bend point to

[1] A table of factors for converting non-SI units of measurement to SI units is presented on page vii.

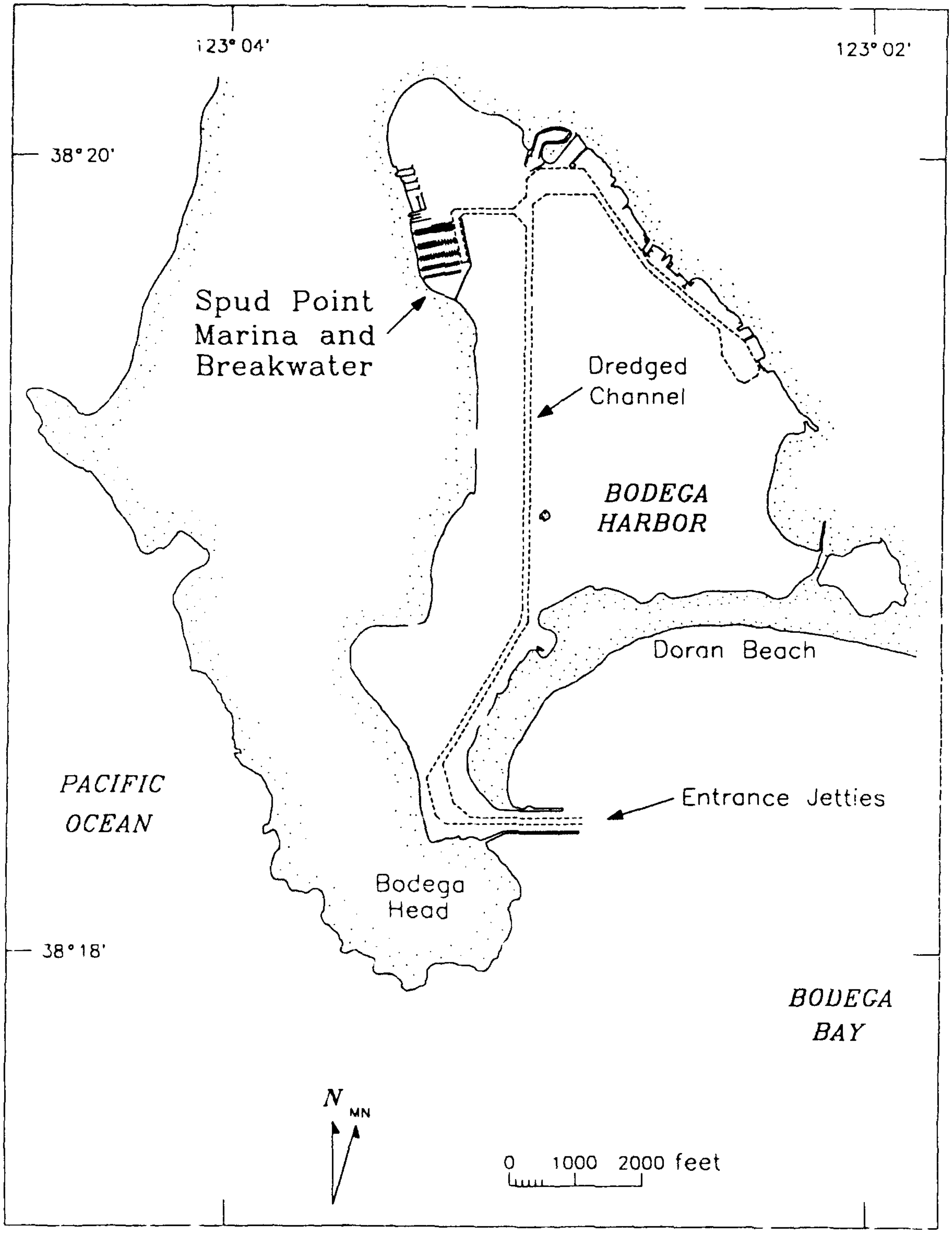


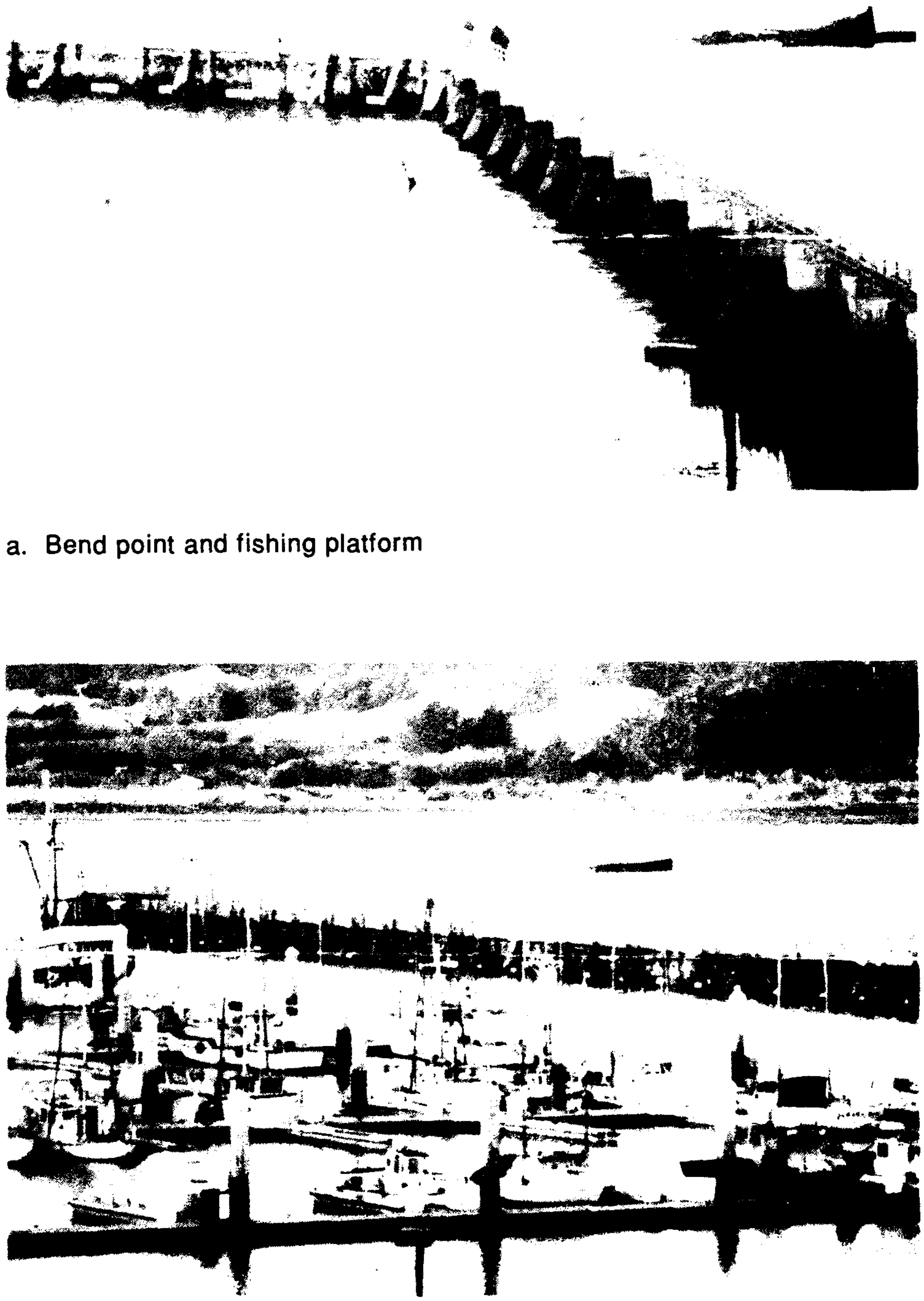
Figure 1. Project location map

Figure 2. Oblique v:ew of Bodega Harbor

facilitate fishing and pedestrian access. A metal walkway connects the breakwater to the shore at the southern end. Angled prestressed concrete batter piles along the marina side of the structure are embedded into the cap beam to form bents, giving increased lateral support. The top of the cap beam is located at an elevation of +8 ft mllw datum. The baffle panels extend from the bottom of the cap beam down to -1 ft mllw. Since the breakwater is open between support piles below the baffle panels, the structure allows a varying degree of resistance to circulation through the marina, depending on the stage of the tide. Figure 3 is an oblique aerial photo of the marina and breakwater looking in an easterly direction, taken October 10, 1985. Spud Point is the promontory and sand flat on the right (south) side of the breakwater; the main Federal channel and narrower access channel are also visible. Figure 4 shows the breakwater at a relatively low tide from a viewpoint on the hillside west of the marina. The floating docks of the marina can be seen, along with the baffle openings, batter piles, and wooden fishing platforms. Figure 5 gives views of the shoreward leg of the breakwater from inside the marina. Figure 6 shows construction details of the inside (marina side) of the breakwater. The tide staff in the center of Figure 6a indicates a water level of about -0.5 ft mllw.

Figure 7 shows a scale site plan of the breakwater and vicinity. (Since no as-built plans or surveys of the breakwater have been available for this study, the site plan was derived from plans found in the Corps' General Design Memorandum (GDM), (US Army Engineer District (USAED), San Francisco 1981) prepared by US Army Engineer District, San Francisco (SPN), aerial photographs, nautical charts, and Corps' hydrographic survey sheets (USAED, San Francisco 1989). State plane coordinates are indicated on the plan. The accuracy of the scale plan is believed to be good, but must be considered approximate. Dredged channels are indicated by dashed lines. The zero elevation mllw contour from National Oceanic and Atmospheric Administration (NOAA) chart 18643 (US Department of Commerce 1986) is shown as a solid line enclosing tidal flat zones indicated by hatching. Selected soundings (mllw datum) from SPN's Bodega Harbor condition survey of October 12-16, 1989 (USAED, San Francisco 1989), are also shown. The selected depths are taken from across-channel lines of Fathometer soundings, using the deepest and shallowest (on the breakwater side of the channel) depth from each line. Chart 18643 does not show the breakwater, but soundings shown for the area between the current breakwater location and the main channel range from -5 to -1 ft mllw. North of the marina, depths are generally slightly deeper than between the breakwater and the main channel. The significance of the bathymetry in Bodega Harbor and especially near the breakwater will be addressed later in this report.

Figure 3. Oblique view of Spud Point Marina and breakwater



a. Bend point and fishing platform

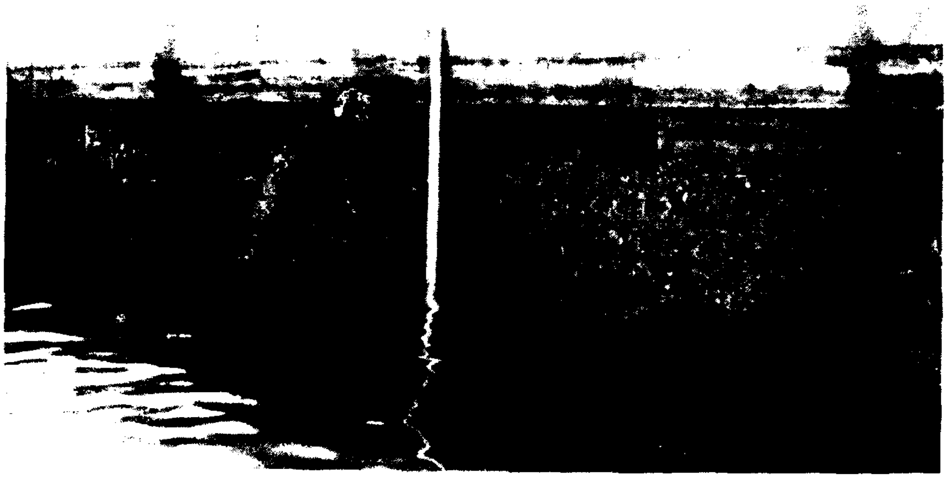
b. Seaward leg of breakwater with floating marina docks in foreground

Figure 4. Breakwater at low tide

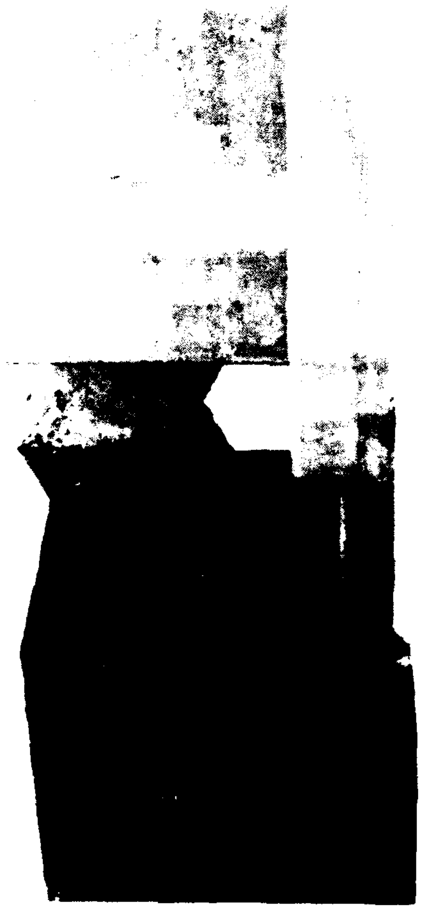
a. View from fishing platform towards walkway

b. View from shore inside marina

Figure 5 Shoreward leg of breakwater



a. Tide staff indicating water level of -0.5 ft mllw



b. Batter pile embedment into cap

Figure 6. Breakwater details

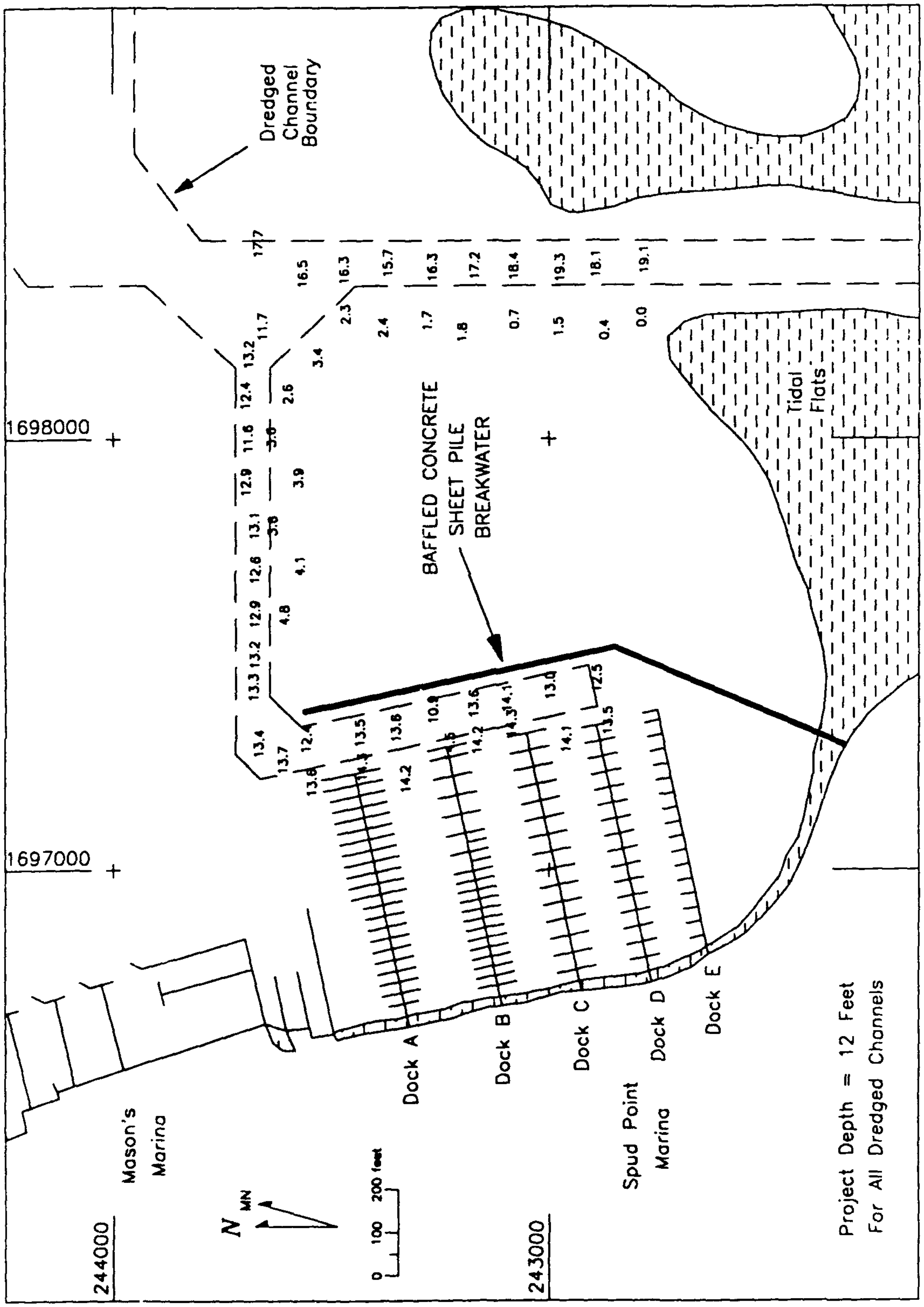


Figure 7. Detail map of marina and breakwater

Project History

The US Army Corps of Engineers has constructed and maintained general navigation facilities in Bodega Harbor since 1943. A recommended plan for improvement of the navigation facilities in Bodega Bay was authorized by the Rivers and Harbors Act of 1965 (PL 89-298), 89th Congress, 1st Session. This plan recommended construction of a 4,500-ft-long earthen breakwater protecting a marina on the east side of Bodega Harbor. Further studies conducted in the 1970's indicated that the plan had become infeasible and unacceptable to local interests. Therefore, a revised project plan was developed. This plan featured a concrete pile baffled breakwater and marina at Spud Point. Plans and specifications for this plan were completed by SPN and the project was included in the President's Fiscal Year (FY) 1984 Civil Works budget as a new construction start. However, because of the unresolved cost-sharing policy for new construction starts, construction funds were not appropriated in FY 1984. The local sponsor (Sonoma County Regional Parks) chose to construct the project with their own funds rather than delay the project, thereby in effect making a 100-percent up-front cash contribution. The final design prepared by Moffatt & Nichol, Engineers, Long Beach, CA, was fundamentally the same as the prior Corps design. The project was constructed between December 1984 and August 1985 and was found by the Corps to be designed and constructed to meet the criteria of the authorized Federal Project. In accordance with ER 1130-2-307, paragraph 13 (US Army Corps of Engineers 1968), SPN proposed (letter dated 8 August 1985 to the South Pacific Division Commander) that the responsibility for the maintenance of the project (breakwater and access channel) be transferred to the Corps. The Director of Civil Works concurred (letter dated 15 November 1985) with the District's proposal, and Sonoma County Regional Parks was notified in 1986 that the Corps would be responsible for project maintenance.

2 Design Parameters

Physical Environment

Tides

Tides in Bodega Harbor are of the mixed type, normally having two high and two low waters per day. The magnitudes of the two highs or two lows (or both) are unequal. The tidal elevations at the north end of Bodega Harbor, where the breakwater is located, vary significantly from those at the harbor entrance. The tidal ranges between mllw and mean higher high water (mhhw) are 5.7 ft and 4.1 ft at the entrance to Bodega Harbor and the north end of the harbor, respectively, indicating that the Bodega Harbor tidal wave amplitude is reduced relative to that of the open coast. A comparison of the tidal elevations at both locations (US Department of Commerce 1964) appears below:

	Bodega Harbor Entrance ft mllw	**North End of Bodega Harbor ft mllw**
Highest estimated tide	8.5	6.5
Mean higher high water	5.70	4.10
Mean high water	5.00	3.40
Half tide level	3.10	2.05
Mean low water	1.20	0.70
Mean lower low water	0.00	0.00
Lowest estimated tide	-2.5	-2.0

The mhhw, 4.1 ft, and the highest estimated tide (HET), 6.5 ft, were critical to the design of the breakwater cap elevation. Predicted tidal elevations for the entrance to Bodega Harbor, which are useful in the planning of field monitoring, can be calculated using the tidal differences and constants for the Bodega Harbor Entrance and San Francisco (Golden Gate) reference station predictions given in NOAA tide tables.

Currents

No quantitative data concerning currents within Bodega Harbor were available prior to the design of the structure or this Monitoring Completed Coastal Projects (MCCP) study. The GDM states that: *observations by the marina operator in the Spud Point area indicated that the general circulation pattern is north-to-south in that area. Remote sensing data indicate that this circulation pattern is north-to-south and then easterly by Spud Point, and that the area where the project is proposed is kept relatively deep by this circulation pattern. The continuous maintenance of the depth in this area is also evidenced by the results of hydrographic surveys which show that except for the existing Federal channel and turning basins, the Spud Point area has been and continues to be the deepest area in the harbor.* No explicit consideration of the currents' effects on the structure, such as scour potential or wave/current interaction, was included in the breakwater design. However, the structure's effects on currents and, consequently, on flushing and water quality in the marina were an important consideration in the decision to use a permeable baffled structure.

Winds

No measured wind data are available specific to the Spud Point site; however, winds are observed daily at 3-hr intervals at the USCG station on the Doran Point peninsula near the entrance to Bodega Harbor. A wind rose from the GDM, based on 7 years of observations between 1972 and 1979, is shown in Figure 8. The anemometer location is depicted on Figure 9, along with two of the critical wave-generation fetches considered in the design wave calculations. On the rose, the length of the bars corresponds to the number of days per year, on average, that winds fell into those range bins. Winds from the fourth windspeed range, "calm," from 0 to 5 mph, cannot be given a meaningful directional classification. The rose shows that Doran Point winds most frequently come from the northwest and that the strongest winds are generally from northwesterly directions also. However, windspeeds from the south to east quadrant are occasionally high, with maximum speeds of 40 mph from all directions between east and south-southeast, and 46 mph from the south. The Doran Point anemometer record was considered to be adequately representative of Spud Point winds for design purposes, and was used without adjustment for calculation of design waves at the marina site.

Waves

Design wave characteristics were determined using forecasting procedures found in the Coastal Engineering Research Center's (CERC's) *Shore Protection Manual* (SPM) (US Army Engineer Waterways Experiment Station 1977). Pacific Ocean waves do not penetrate as far as Spud Point, so only locally generated waves needed to be considered. Boat wakes within the marina were expected to be negligible, since speeds are restricted to

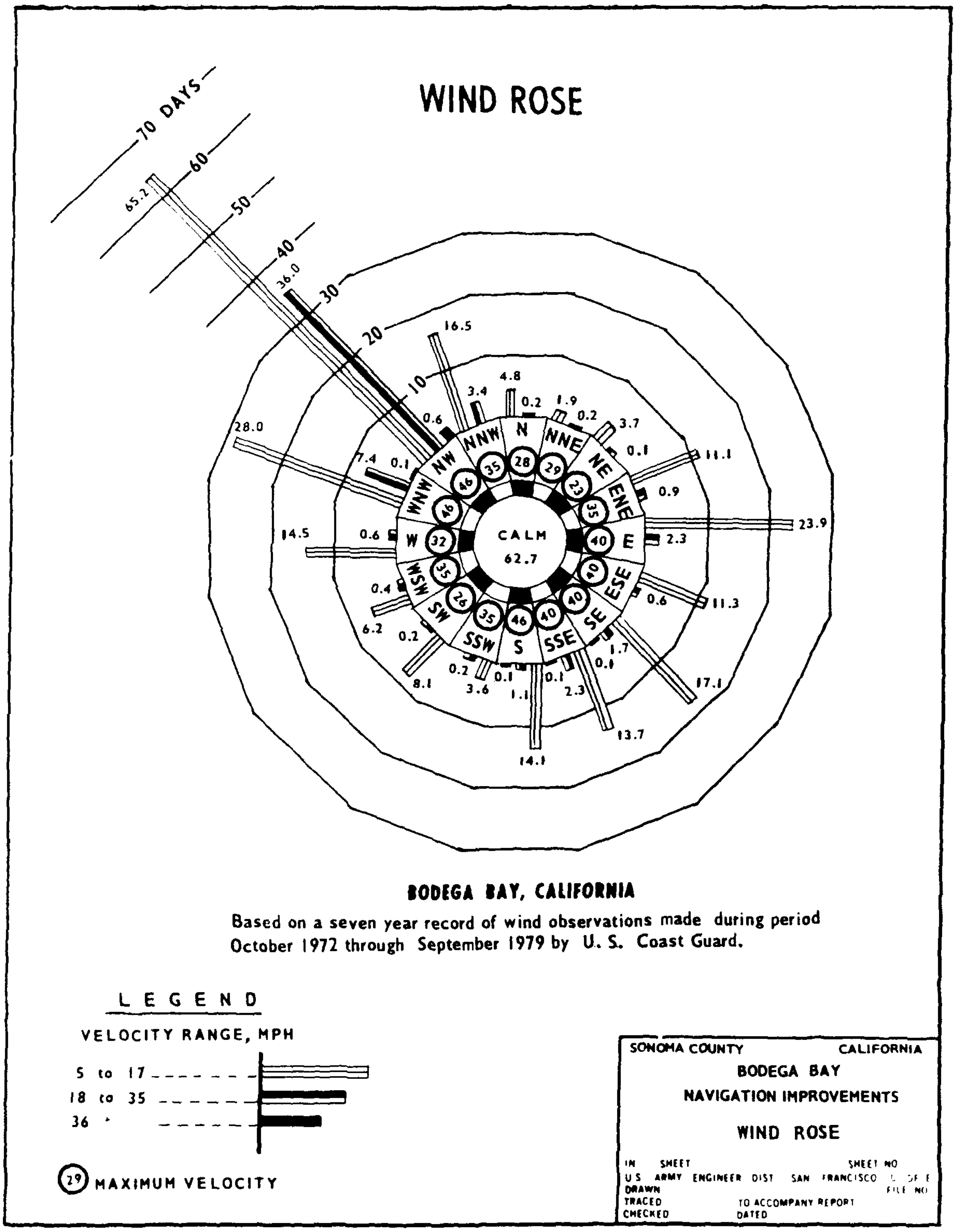


Figure 8. Long-term wind rose for Doran Point USCG anemometer

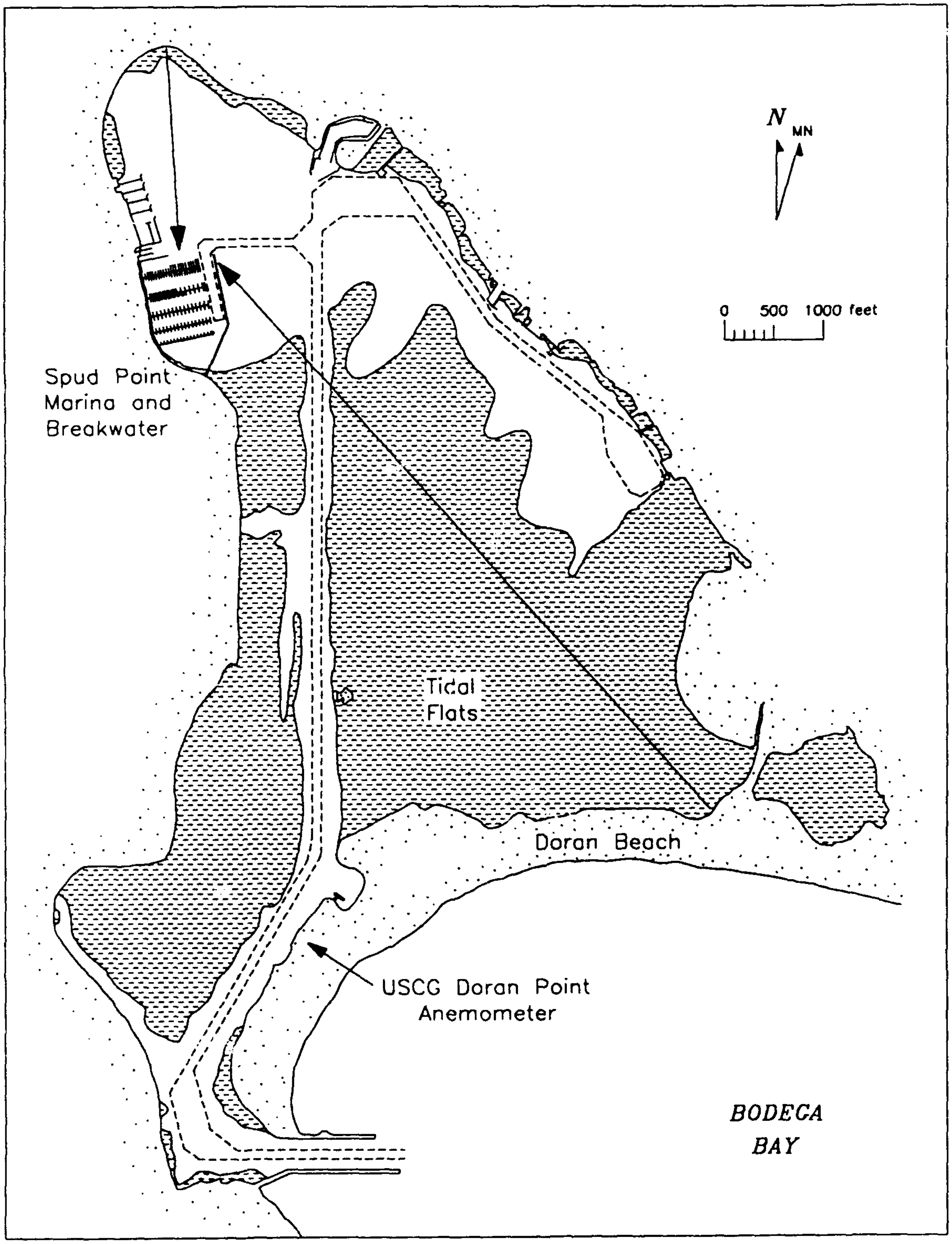


Figure 9. Critical wave generation fetches for breakwater design

5 mph in such areas. Winds from northwesterly to southerly directions were considered unimportant for wave generation, due to the sheltering of the marina site by the Bodega Head peninsula, and the lack of long fetches in those directions. From a review of the wind rose versus fetch orientation, fetch length, and water depths, two critical fetches were selected for calculation of waves (see Figure 9) incident to the proposed marina site. The SPM shallow-water forecasting curves were used assuming a still-water level (swl) of 4.1 ft, the mhhw elevation for northern Bodega Harbor. Forecast waves for HET conditions were not given in the GDM.

For the northerly fetch, a windspeed of 30 mph, fetch length of 2,000 ft, and depth of 10 ft give a significant wave height H_s of about 0.9 ft and a significant period T_s of about 1.8 sec. The SPM recommends use of H_1, the height of the average of highest 1 percent of all waves (= 1.67 H_s), as the design wave height for rigid structures. Therefore, the northerly fetch could produce a design wave height of about 1.5 ft. Waves of this height were expected to occur infrequently and would be about the maximum height tolerable in a marina. Also, a floating work dock was expected to be placed along the northern part of the marina for use by commercial fishing boats. Since the floating dock would provide some wave reduction in addition to that already provided by structures at Mason's Marina just north of the site, it was felt that no breakwater structure would be needed for the north side of the marina.

The wave forecast for the southeasterly fetch used a windspeed of 40 mph, fetch length of 7,500 ft, and depth of 5 ft. Resulting H_s and T_s were 1.5 ft and 2.5 sec, respectively, giving a design wave height H_1 of 2.5 ft. It was felt that a protective structure would be needed to reduce waves approaching from that direction.

Other factors

Bottom sediments and bathymetry were considered with respect to dredging of the marina and access channels, and the foundation conditions for the structure. Earthquake-related damage (displacements of the structure) is considered to be likely for earthquakes of Richter magnitude 6 or greater. Extensive damage, requiring replacement of the structure, is expected for earthquakes of magnitude comparable to the 1906 event (Richter magnitude 8.3 at the epicenter located 25 miles southeast of Bodega Harbor), which caused large horizontal displacements along the San Andreas Fault in the Bodega Bay vicinity. Tsunami damage is also considered possible. No special design features were incorporated into the breakwater to prevent or reduce either earthquake or tsunami damage.

Breakwater Design

Introduction

Selection of the Spud Point site and recommended breakwater design involved many other considerations besides those covered in the preceding sections, including costs over a 50-year design life span and environmental impact. Several alternative breakwater types and sites were evaluated. Configuration and maintenance of dredged channels and dredge disposal options played an important part in the selection of the recommended plan. The GDM describes the alternative plans and covers the selection process in detail. This report only discusses those elements of the recommended plan's design relevant to the MCCP effort and of a more general coastal engineering interest. Although the final design and construction were not under Corps' supervision, the resulting structure was judged by SPN to be functionally the same as that presented in the GDM. From site visits, it has become apparent that the constructed breakwater is not identical in all respects to the Corps' plan; for example, the spacing of batter piles and cap configuration are different. The overall length and orientation of the structure are similar, if not identical, and cap and baffle bottom elevations are the same as the Corps' design. Appendix A shows the Corps' design plan from the GDM. Since many details of the final design process and the as-built drawings have been unavailable for this MCCP study, the following paragraphs will discuss the design described in the GDM.

Breakwater type, configuration, and loading

The breakwater is a baffled, pile-supported, reinforced concrete structure. Its 1,306-ft length is made up of a 576-ft-long shoreward leg oriented in a south-southwest to north-northeast direction, and a 730-ft-long seaward section oriented in a north-northwest to south-southeast direction. Length and orientation were determined considering wave approach direction and the desired marina location and berthing layout. The GDM indicates that wave diffraction around the seaward end of the breakwater was examined as part of the length and orientation design, using procedures outlined in the SPM. The concrete structure includes baffle panels, pile bents, intermediate piles, and a continuous cap beam. Baffle panels were designed as simple beams supported by the vertical bent piles and intermediate piles. Their design considered bending stresses due to wave loads and panel dead load. In the most seaward part of the constructed breakwater, both ends of baffle panels are supported by vertical piles in bents with batter piles. The batter piles were designed for axial load due to wave loads and bending stresses due to dead load. The vertical bent piles were designed for axial load due to dead and wave loads and bending stress due to wave loads. The intermediate piles are similar to the vertical bent piles. The cast-in-place cap was designed as a continous beam for vertical loads supported by the bents and intermediate piles, and for horizontal loads supported by the bents alone. It is not clear whether uplift forces due to waves striking the underside of the cap were considered in the design. The GDM states that wave forces were calculated in accordance with Engineer Manual (EM) 1110-2-2904, "Design of Breakwaters and Jetties" (US Army Corps of Engineers 1986), as applicable to non-breaking, breaking, and broken waves. Breaking conditions were determined on the basis that the average ratio of the depth of water in which a wave breaks to the wave height at the point of breaking is approximately 1.3. Wave forces were calculated by the Sainflou method for non-breaking waves, and by Minikin's method for breaking waves, both of which are presented in EM 1110-2-2904 (US Army Corps of Engineers 1986).

Cap elevation

The crest height of the breakwater (top of cap elevation) was arrived at using methods detailed in EM 1110-2-2904 for the case of non-breaking waves forming a standing wave (clapotis) due to reflection off the wall. The design wave height, 2.5 ft, was used along with an swl (= mhhw elevation) of 4.1 ft plus a wind setup of 0.4 ft in an equation which EM 1110-2-2904 refers to as the modified Sainflou method or the Miche-Rundgren method. The method gives the superelevation (above the swl) of the mean level of the clapotis. The crest of the clapotis was calculated to occur at about +7.7 ft mllw, determined as this mean level plus the design wave height. The cap elevation was consequently set at +8.0 ft mllw. For the HET swl of +6.5 ft mllw, the calculated clapotis crest elevation was +10.2 ft, an overtopping condition. Since HET conditions were considered an infrequent occurrence, and the distance between the breakwater and the nearest berths was considered likely to be sufficient to result in dissipation of overtopped waves, it was felt that the +8.0-ft cap elevation would be adequate.

Baffle panel bottom elevation

As previously described, the baffled breakwater uses baffle panels which extend only part of the way from the cap to the harbor bottom. Lowering the panel bottom elevation decreases the wave transmission, but also decreases the flushing potential of the marina. The design approach was to find the highest panel bottom elevation possible, given the allowable wave transmission criteria. Since there is no published Corps' design guidance for this type of breakwater, results presented by Wiegel (1960, 1964) in his landmark book, *Oceanographical Engineering*, were used. Wiegel developed a nomograph from a first-order transmission expression based on wave power for the case of an infinitely thin, rigid barrier extending from above the water surface to some distance below the water surface. The theoretical approach assumes monochromatic waves over a flat bottom and does not consider the effects of supporting piles. A detailed description of the theoretical development and laboratory tests performed for comparison with the theory's predictions are given in an earlier journal article (Wiegel 1960). Figure 10 shows the version of Wiegel's nomograph given in the US Navy's Coastal Protection Design Manual (US Navy 1982). The nomograph relates the transmission coefficient K_t (ratio of transmitted wave height to incident wave height) to the ratio of the baffle panel penetration depth to the stillwater depth h/d_s for a range of relative depths. The relative depth is the local ratio of depth to wavelength d/L occurring at the structure. The relative depth curves on the nomograph are labeled with d/L in conformance with the nomenclature of the SPM. For example, one might compute the deepwater wavelength L_0 using the design wave period, then compute an equivalent deepwater relative depth d_s/L_0 (using the stillwater depth at the structure) then find local relative depth d/L using Table C-1 of the SPM (1977) (which relates d/L and d/L_0). Given a design stillwater elevation, mudline elevation on the structure, and incident wave height, the nomograph can be used to determine the required baffle bottom elevation corresponding to a transmitted wave height criterion. The GDM states that with the bottom of the baffle panels set at -1.0 ft mllw, the transmitted wave heights would be about 1 ft under design conditions for mhhw and HET stillwater levels (K_t of about 0.4 in Figure 10), and lower than 1 ft for lower tidal stages. Although not stated in the GDM, apparently the mudline elevation on the structure was assumed for transmission calculations to be at about -7 ft mllw, which was also assumed in the final design by Moffatt & Nichol.

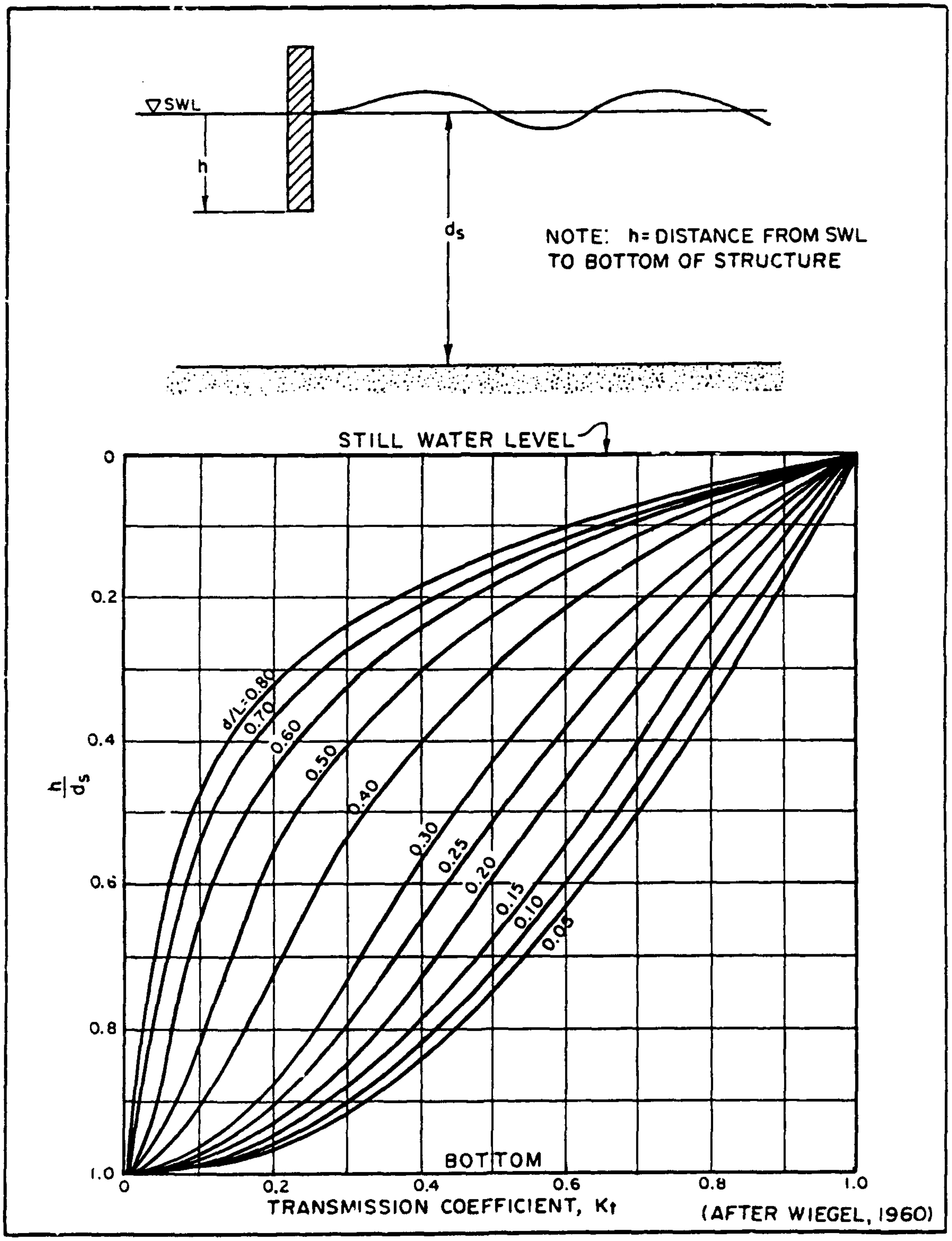


Figure 10. Nomograph for wave transmission coefficient

3 Monitoring Effort

Objectives

The overall objective of the monitoring effort was to determine whether the unusual baffled breakwater is performing as designed. The specific objectives established prior to the field effort were as follows:

a. Evaluate the flushing characteristics of the baffled breakwater.

b. Measure the transmitted wave produced by waves of various heights and periods impacting the breakwater at differing tide levels.

c. Within the limitations imposed by the monitoring schedule and the primary requirement to satisfy objectives *a*. and *b*., evaluate the structural integrity of the baffled breakwater.

Criteria for Success

The criteria for evaluating the success of the baffled breakwater design are given below for each specific monitoring objective:

Currents

The current under the baffles of the breakwater is measurable and indicates a general pattern of flow from the inside to the outside of the marina. This will show that the design has allowed the natural north-to-south flow in the area to continue, preventing stagnation of water in the marina.

Waves

The transmitted wave heights measured behind the breakwater for various incident wave heights will be less than or equal to the transmitted wave heights predicted by the nomograph of Figure 10. This will indicate that the design methodology was appropriate and applicable to this project.

Structural integrity

No significant distress of the breakwater has occurred. This includes major misalignment, differential settlement, and cracking or spalling of the concrete deep enough to expose the reinforcing steel to the environment. Lack of significant distress will indicate that the design is structurally capable of resisting the forces applied to it without experiencing destructive movement.

Prototype Data Collection, Analysis, and Interpretation

Overview

A field monitoring effort was conducted at Spud Point Marina and breakwater during 7-18 March 1988. The monitoring plan developed jointly by CERC and SPN originally called for two field data collection site visits to allow comparison of data to determine if a scour problem might be developing. A preliminary examination of results from the March field effort showed no evidence of scour development, and it was concluded that the expense of the second data collection effort was not warranted.

Current profile measurements

Current profile measurement stations A through F were selected at sites along the breakwater and north side of the marina as shown in Figure 11. Sites A through D were located adjacent to baffle openings; the figure gives the sequential baffle opening number (numbers start at the first opening at the north end of the breakwater) for these stations. A series of hourly measurements of horizontal velocity were obtained over the 25-hr period (one tidal cycle) from approximately 11:00 Pacific Standard Time (PST) on 9 March through 12:00 PST on 10 March, with three readings taken over the water column at each site. For stations B, C, and D, the readings were taken at 2 ft above the bed, at the center of the baffle opening, and at 1 ft below the bottom edge of the baffle panel. At station A, only a single reading was taken during each round of sampling, due to the small height of the baffle opening there. At stations E and F, the bottom

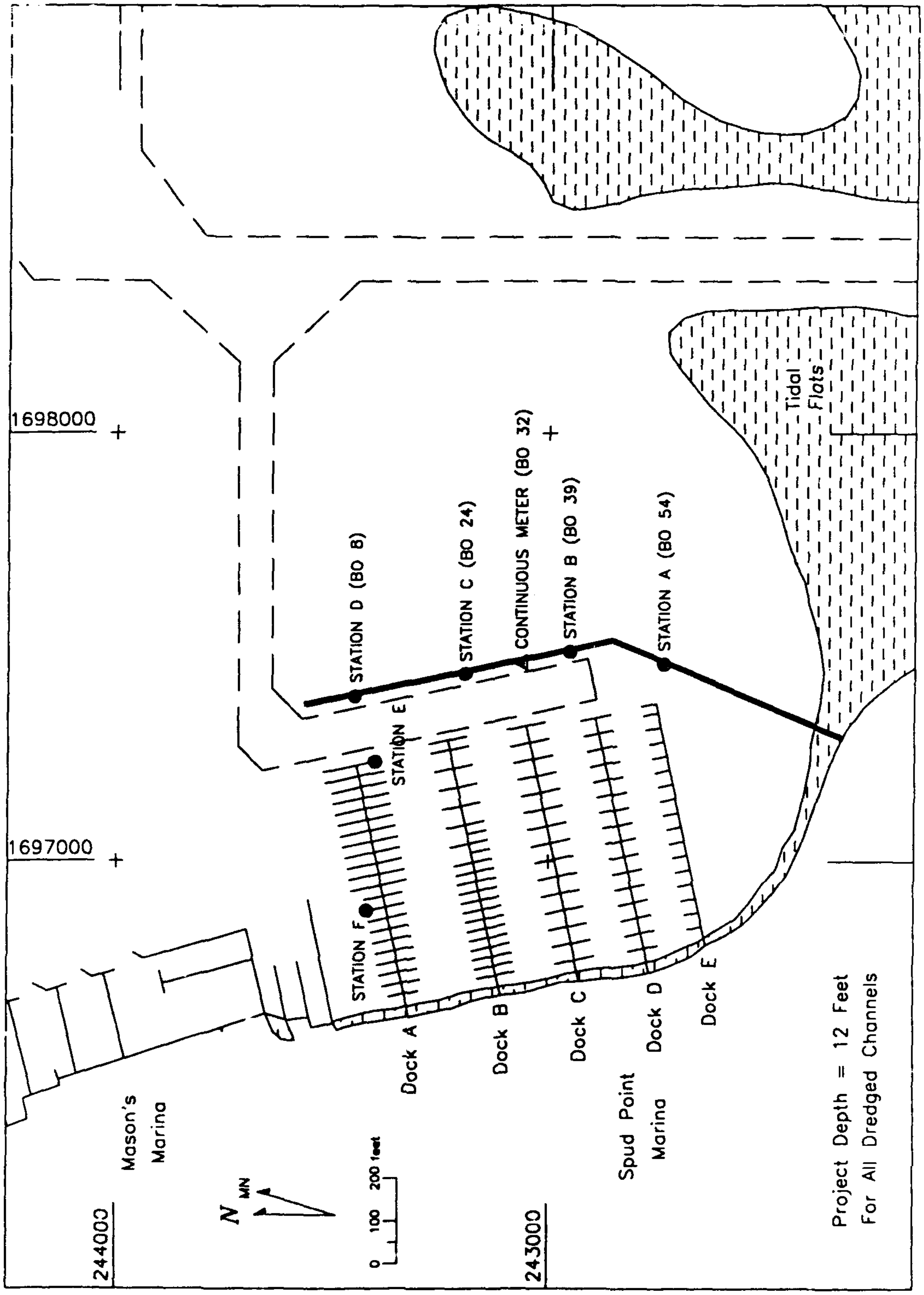


Figure 11. Current measurement stations

reading was 2 ft above the bed, the middle reading was at mid-depth, and the top reading was at 3 ft below the water surface. Current measurements were obtained using a Teledyne-Gurley direct reading current meter (model No. 665). The speed threshold is 0.2 ft/sec for this exposed-cup impeller meter. Measurements were made from a skiff tied alongside the floating docks (stations E and F) or tied to the breakwater. Tide stage readings were obtained hourly from a tide staff affixed to the breakwater. Wind observations were recorded by the Doran Point USCG station every 3 hr.

Figure 12 shows measured tide, winds, and station E currents versus time during the profile study. The tide plot is a smooth curve fit to the observed data. The wind data are shown as vector velocities, as are the current data. Figure 12 allows a comparison of the currents with the major driving forces, and also shows the depth variation. As can be seen in the figure, the winds were relatively strong and steady from the northwest, but the current direction appears to be more strongly dependent on the tide than on the winds. Current speeds lower than the threshold of the meter (no vector plotted in the figure) occur around the times of higher high water and lower low water. Current magnitudes are generally greatest at the top of the water column, and the flow is complex and somewhat stratified as indicated by the variation in direction between readings at different levels for corresponding times. Figure 13 shows the measured mid-depth current vectors for all of the stations together. Stations A - D along the breakwater showed minor depth variation, so the mid-depth values (center of baffle opening) are representative. This figure gives an impression of the site's general circulation pattern. The stations along the breakwater had a predominantly north-to-south flow throughout the 25-hr study. The data indicate that flow generally entered the marina through the breakwater during the rising tide, and seems to have exited along the northwest part of the marina. During the falling tide, the flow more closely paralleled the breakwater, but was still generally north to south. Observations near the landward end of the breakwater would be useful to further identify the circulation pattern.

Continuous current measurements

A self-contained, recording current meter was used to obtain a more detailed time-dependent record of currents passing through the breakwater over several tidal cycles. The Interocean model S-4 electromagnetic current meter was deployed just inside the breakwater at a baffle opening, as shown in Figure 11. The meter was mounted on a weighted bottom pod so that it was about 2 ft above the bottom. The adjacent baffle opening was about 6 ft high. Currents were sampled twice each second for 5 min out of every 10 min. The meter computed and stored a vector-averaged horizontal velocity for each 5-min sampling period. Hourly depth readings were also obtained by means of a pressure sensor incorporated in the meter. Figure 14 shows current and tide stage data obtained by the meter versus winds observed at Doran Point from 15 to 18 March. Note that

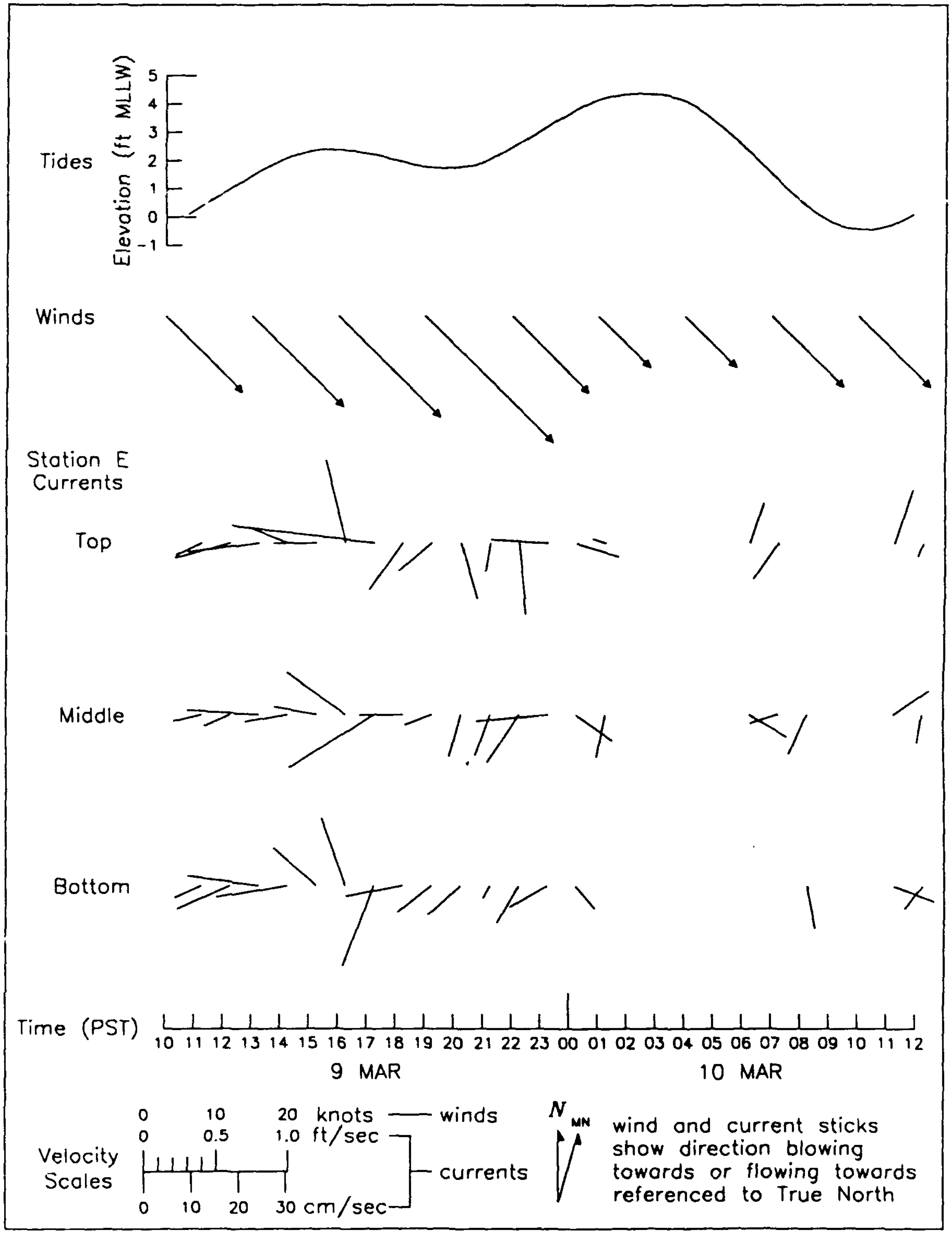


Figure 12. Measured tides, winds, and station E currents versus time, 9-10 March 1988

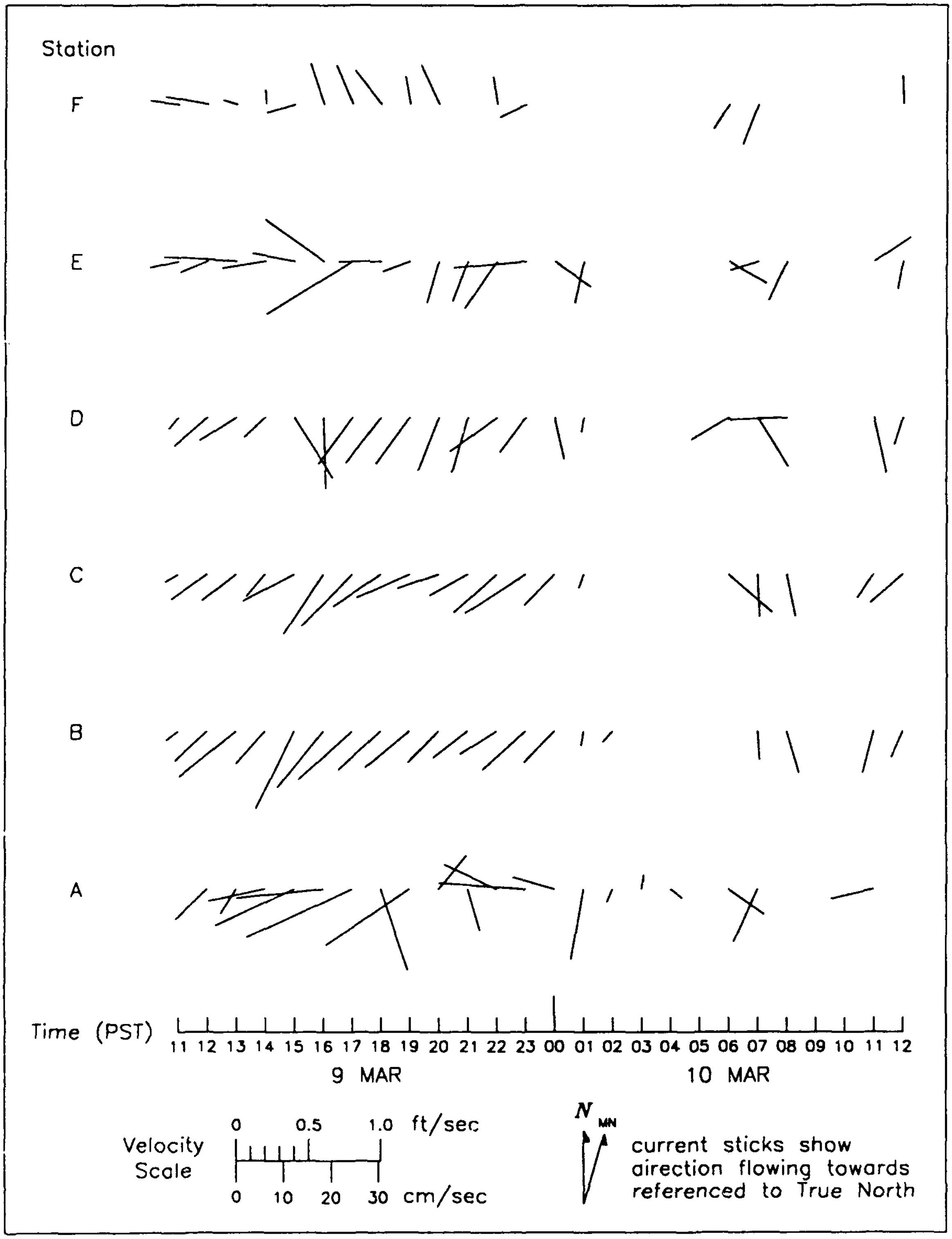


Figure 13. Measured mid-depth currents (all stations), 9-10 March 1988

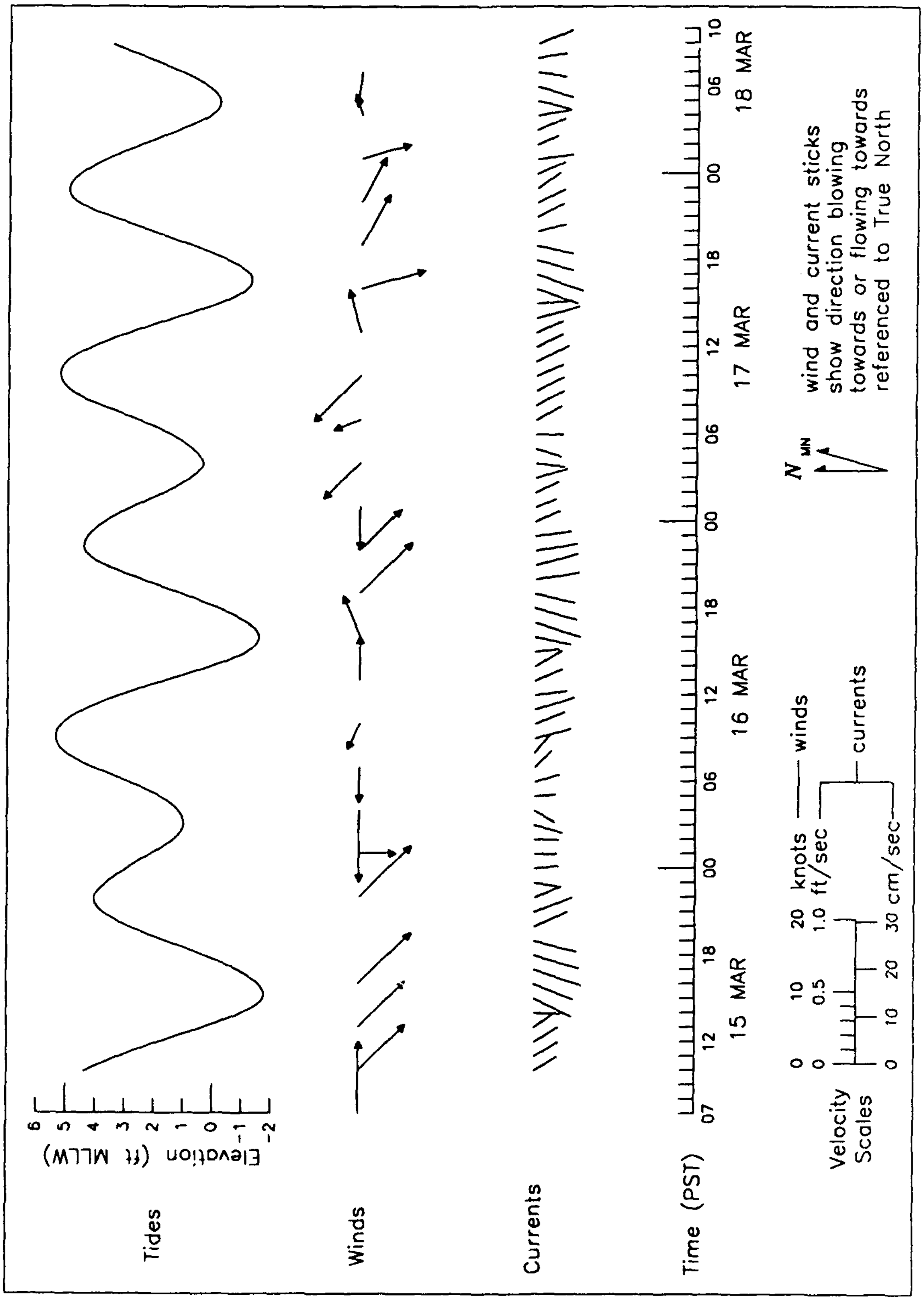


Figure 14. Measured tides and winds versus continuous currents, 15-18 March 1988

only hourly current vector data are shown, extracted from the more detailed dataset. The tide plot is a smooth curve fitted through the data obtained by the S-4 meter. Note that the tide height and the wind and current velocity scales are the same on Figures 12 and 14. The winds were much stronger during the period shown on Figure 12 than that of Figure 14, but the tide variation was greater during the period of Figure 14 than that of Figure 12. However, the measured currents were stronger during the earlier profile study. If the measurements obtained using the two different current meters and sampling procedures are indeed equiva-lent, this indicates that current strength is strongly dependent on winds as well as tide stage. Figure 14 shows that the current direction was gener-ally parallel to the seaward leg of the breakwater, and the component of velocity through the breakwater was usually weak compared to that along the breakwater. It is remarkable that the current never deviated from a generally north-to-south pattern throughout the deployment period.

No water quality data were obtained for this MCCP study. Some surface debris tends to accumulate along the inside of the southern part of the breakwater at high tide when the baffles are submerged. Although not necessarily indicative of good water quality, the marina operator reports that fishing from the breakwater has become popular.

Visual inspection

The breakwater was visually inspected both from a boat traveling along it at low tide and by walking the cap on 10 March. No notable evidence of misalignment, movement, spalling, or other structural problems was found. A second brief walking inspection was performed on 23 February 1989. Very narrow hairline cracks across the full width of the top of the cap at nearly all the bent locations (where there is a change in cap width) were observed during the second inspection. Figure 15 shows a typical crack. The key and pencil indicate the points of the "W"-shaped crack. It is not known how deep the cracks penetrate, and they are so small that they went unnoticed during the March 1988 inspection. Mr. Walter Hurtienne[1] of Moffatt & Nichol's Long Beach, CA, office (project engineer at Moffatt & Nichol during the Spud Point breakwater work) indicated that the cracks appeared immediately after the cap was cast, and that they are probably shrinkage related. Future inspections should reexamine these cracks to see if they are growing wider and to look for evidence of corrosion in the reinforcing steel.

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[1] Personal Communication, 4 March 1990, Walter Hurtienne, Project Engineer, Moffatt & Nichol, Engineers, 250 Wardlow Road, Long Beach, CA.

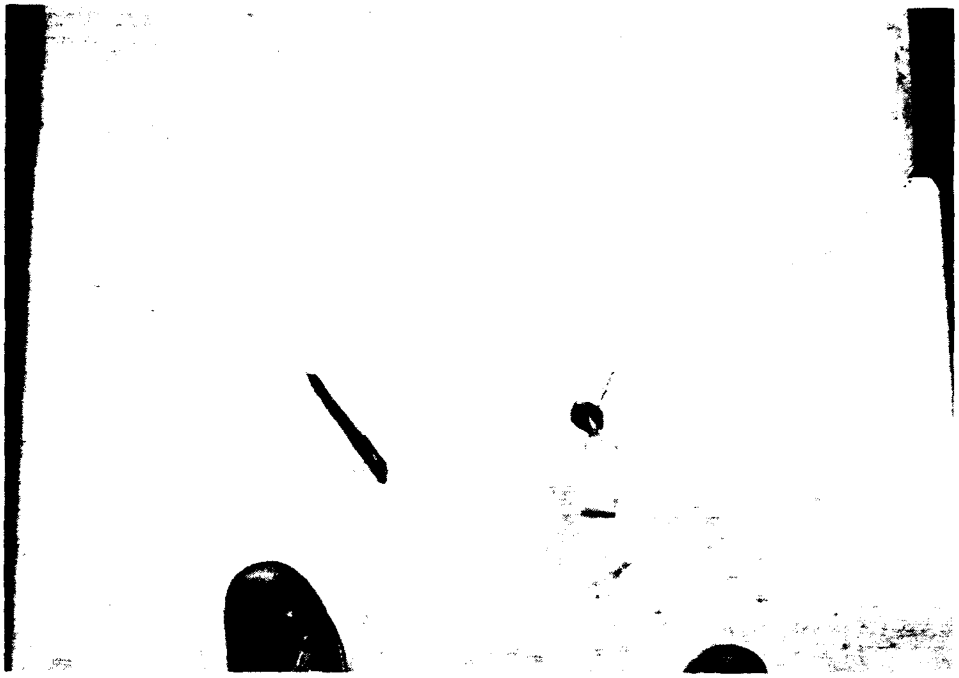


Figure 15. Typical hairline crack in breakwater cap

Side-scan sonar survey

A side-scan sonar survey was performed on 16 March to examine the breakwater for evidence of scour development. An EG&G model 260 side-scan sonar was used to image both sides of the seaward leg of the breakwater. The landward leg is situated in water too shallow for a side-scan survey (except possibly at a very high tide stage). Figure 16 shows a composite of two separate passes along the breakwater. The upper half of the figure is the image from a pass along the seaward side; the lower half is the image from the pass along the marina side. The two images do not overlap, so the breakwater is seen on both images. The darkness of the image varies according to the strength of the reflected acoustic energy. The structure reflected strongly, as did bottom features and slopes. Corresponding points in the two images are noted on the figure. Note that the sonar was able to "see" through the baffle openings, especially during the marina-side pass. Because the marina-side bottom is much deeper, the seaward-side pass did not image bottom features inside the marina. Although there is a spot near the midpoint of the seaward leg which appears to be deeper under the breakwater than elsewhere, the images show no evidence of scour-induced bottom features.

Baffle opening dimensions and cap elevation survey

The elevation of the top of the breakwater cap was spot checked at several locations on 15 March 1988, using conventional land survey instruments, and was confirmed to be at the design level of +8 ft mllw. Baffle

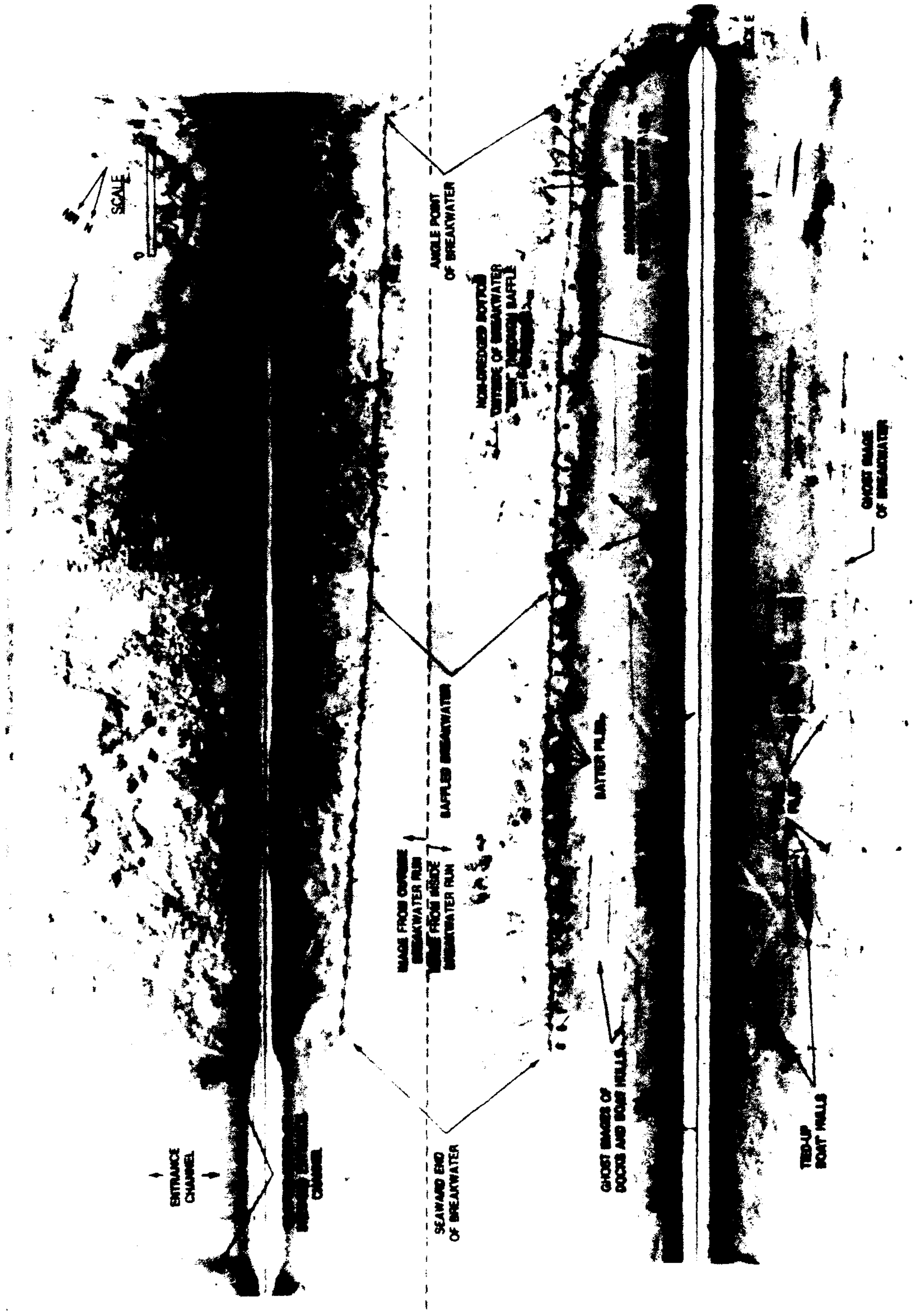


Figure 16. Side-scan sonar images from inside and outside breakwater

opening dimensions were also measured, by lowering an "L"-shaped graduated rod from the cap and hooking it under the bottom of each baffle panel, then lowering further until the rod touched bottom. These elevations were determined by reference to the top of the cap (assumed level at +8-ft elevation). Figure 17 shows a plot of the resulting baffle opening dimensions (dark vertical bars) versus horizontal position along the breakwater. The vertical dimensions are plotted to scale; the horizontal axis is only approximately scaled. The variation in the baffle panel bottom elevations was mostly due to varying accumulations of marine growth on the panels. The mudline elevation along the structure can be interpolated by connecting the bottoms of the baffle opening bars in Figure 17. Onsite observation found the mudline elevation to be deeper than -12 ft mllw at the seaward end and higher than -1 ft mllw at the landward end, where the baffle panel bottoms are buried. There is, however, a shallow gap at the landward end, between the end of the structure and the shore (see Figure 5b) which is open to flow during high tide. The relatively deep spot noted previously on the side-scan sonar image is seen on Figure 16 at baffle opening No. 24. The baffle opening data give no evidence of scour problems, but may be useful for comparison with future datasets. From the figure, the -7 ft mllw mudline elevation used in the wave transmission calculation seems to have been a reasonable choice for a working mudline elevation on the seaward leg of the breakwater.

Wave transmission experiment

Approach and plan. Alternatives considered for direct measurement of the wave attenuation performance of the breakwater included long-term wave gage deployment to observe existing wave conditions, and a short-term experiment to produce waves by means of boat wakes. Although the long-term approach would provide realistic prototype information, high complexity and costs made the approach infeasible. Also, even a long deployment of a year or more could fail to capture enough large wave events to provide a basis for evaluation of breakwater performance. It was felt that an intensive short-term boat wake experiment would guarantee suitable wave conditions and allow a more efficient use of resources. It was recognized that a boat wake experiment would be a compromise effort.

Locals claimed that the wakes of large boats in the area strike the breakwater as incident waves up to about 2 ft in height. The Doran Point USCG personnel claimed their 44-ft-long lifeboats produce waves 4 to 5 ft in height. Thus, an experiment was planned using the Coast Guard boats to generate wakes by running up and down the main Federal channel.

Since the transmission performance and baffle opening design were specified in terms of wave height reduction, the experiment focused on indirect measurement of incident and transmitted wave height using two non-directional wave gages (pressure sensors). The time-varying water surface can be calculated from the measured pressure variation. Gage positions are shown in Figure 18. The outside gage was sited away from the

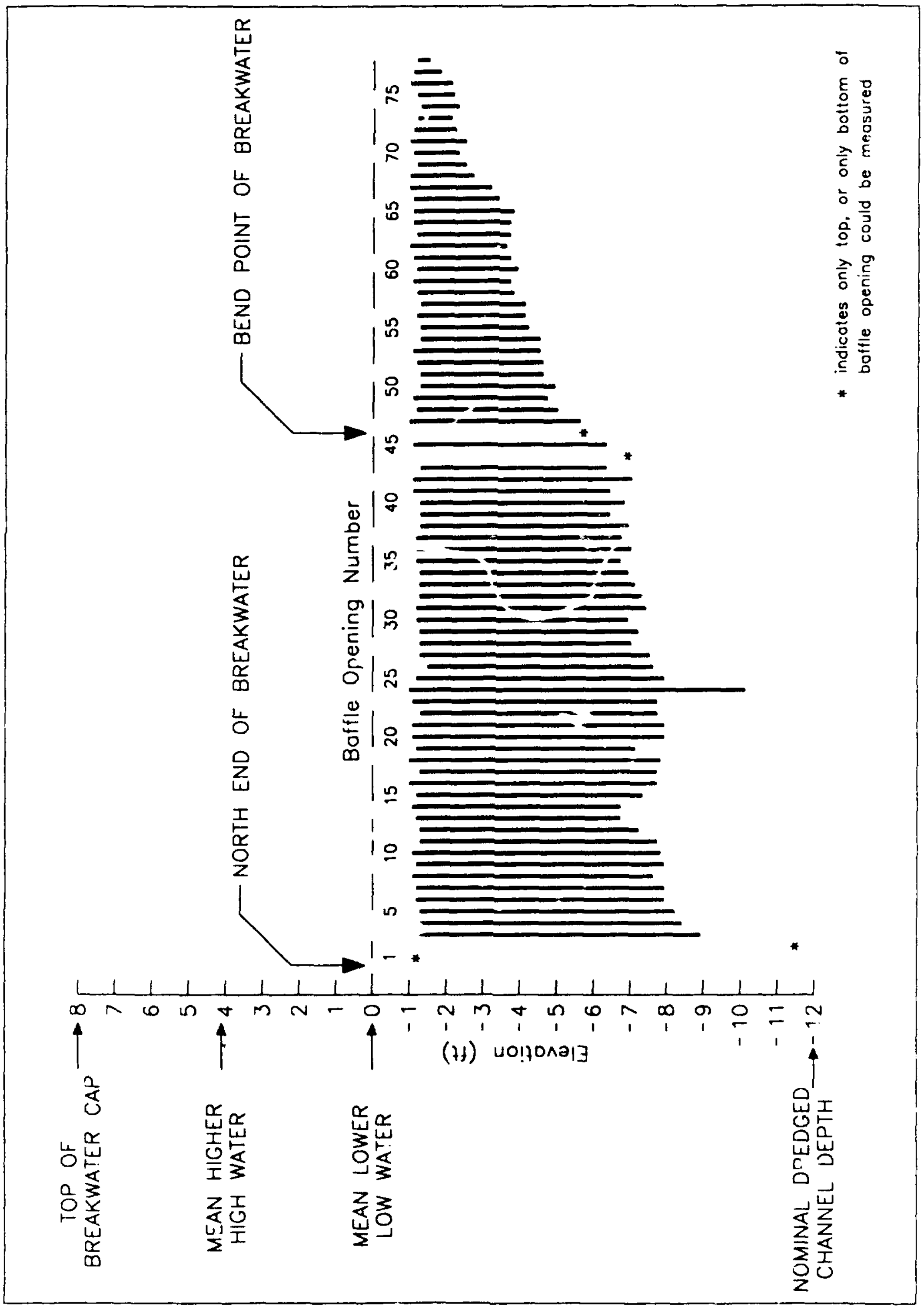


Figure 17. Vertical baffle opening dimensions

highly reflective breakwater, to permit collection of several wave periods of uncontaminated record for each boat run. Assuming shallow-water wave celerity and 4-sec waves, it was estimated that the outside gage would need to be at least 200 ft from the breakwater to permit the passage of five incoming wave crests before reflected waves would return to it and contaminate the record.

Equipment and procedure. The wave measuring system consisted of two Setra model 280E strain-gage-based pressure transducers connected via unarmored cables laid across the bottom to a custom-built serial asynchronous unit (SAU), the output of which was fed to a laptop computer for data storage on floppy disk. Both pressure sensors were mounted at about -2 ft mllw. The inside gage was mounted on a pile near the end of Dock C. The outside gage was attached to a temporary bottom-sitting mount about 240 ft away from the breakwater. Analog outputs from the two sensors were digitized by the SAU at a rate of five instantaneous pressure values per second. The SAU was located adjacent to the marina office building.

Two 44-ft USCG lifeboats were made available for the experiment on 17 March 1988. The experiment consisted of running the boats north or south in the main Federal channel at a constant speed to generate a wake, then turning on the wave measuring system to capture the passage of the wave train. A videotape was made of the experiment. Coordination between the observers on the breakwater, the boats, and the technicians at the SAU was by radio. Initial runs were made by a single lifeboat. Since the generated waves were very small, subsequent morning runs were made using both boats running in staggered fashion, so that the wakes would reinforce each other. The higher high water (+5.3 ft mllw) occurred during the morning runs and the lower low water (-1.4 ft mllw) occurred during the afternoon runs (see Figure 14 for tides, winds, and currents). Increased boat speeds and the added boat were used in the morning runs to try to get larger waves at the breakwater, but unfortunately, the larger waves were also propagating into less-protected marinas on the east side of Bodega Harbor, and complaints forced a halt in the experiment until the low tide in the afternoon. A total of 20 runs were made. The digitized pressure records were examined later at CERC using a commercially available signal processing software package.

Results and interpretation. Only the last three of the morning runs produced visually detectable transmitted waves. During the afternoon runs at lower tide levels, the generated waves appeared to be completely dissipated as they crossed the shallow, relatively flat region fronting the breakwater. Even at higher water there was considerable dissipation of the generated waves, so that the approaching wave train was difficult to distinguish from the wind chop. Although the water inside the marina was more calm than outside, even the largest transmitted waves had very small amplitudes and were hard to see. The most noticeable evidence of transmitted waves was rolling motion of some of the docked boats. The larger generated waves could be clearly detected as they hit the breakwater due

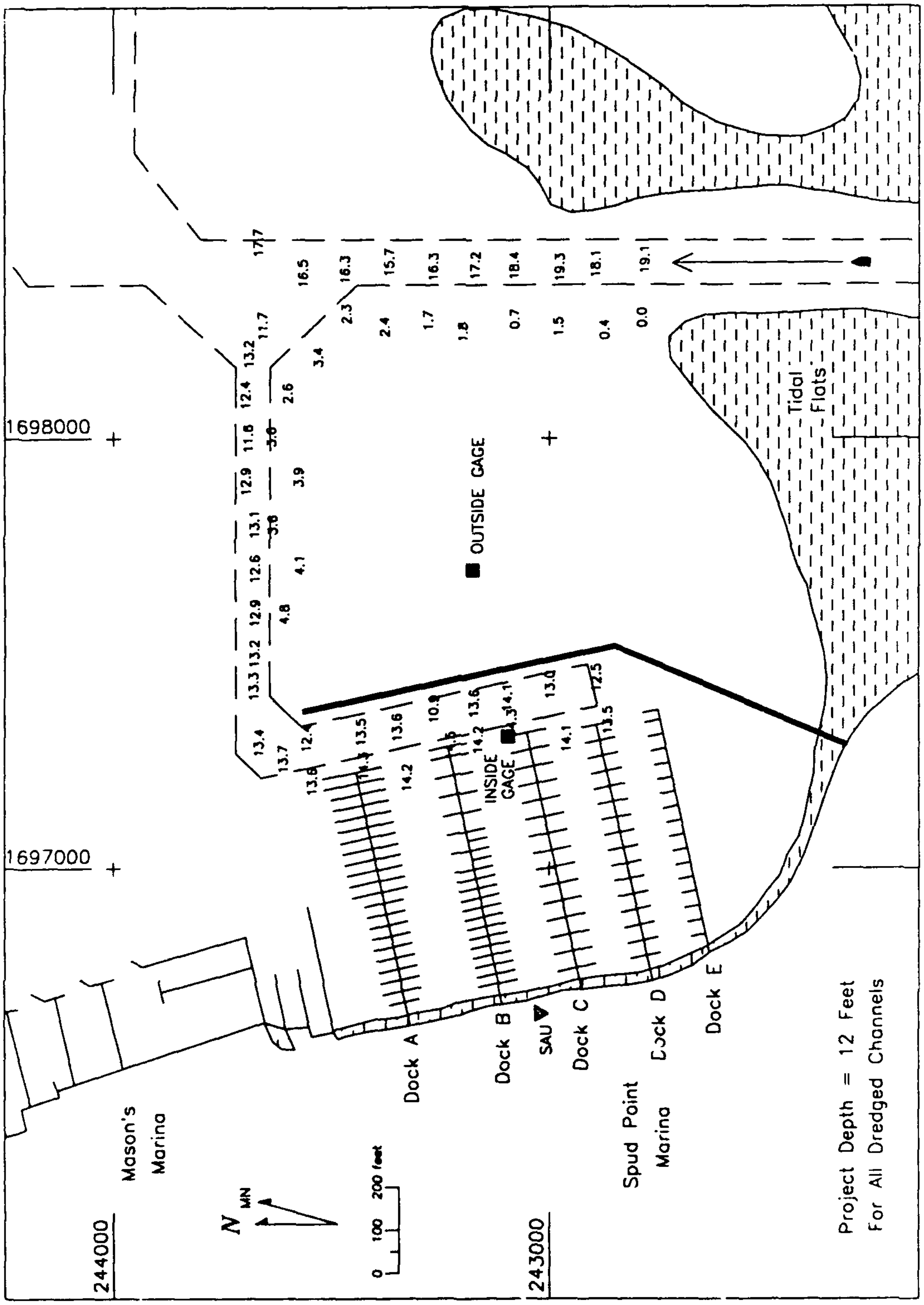


Figure 18. Wave gage locations for boat wake experiment

to the induced vibration and the slapping sound. Incoming wakes were striking the breakwater at an angle of about 45 deg. Unfortunately the transmitted waves from the largest wakes appeared to be more energetic at the north end of the marina on northbound runs, and at the south end of the marina on southbound runs, than at the inside gage location. The largest waves generated were visually estimated to be about 1 ft in height at the breakwater, while the transmitted waves appeared to be no more than a few inches in height. When the pressure records were examined at CERC, it was found to be impossible to distinguish transmitted waves from the background noise level of pressure variation which was primarily due to the small-amplitude, wind-induced surface disturbance.

At this point it was decided that although a more sophisticated approach to analysis of the pressure records might be able to separate the generated waves from the wind waves, the uncertainties involved in any quantitative estimate of measured transmission coefficient from these data would be large, and additional analysis efforts would likely gain little. Clearly the combination of the breakwater and the site-specific factors such as the shoaling over the relatively shallow region fronting the breakwater resulted in substantial wave height reduction for the lifeboat wakes. It is believed that transmission coefficients for the lifeboat wakes were lower than the design condition transmission coefficient of 0.4, but this cannot be verified.

The long-term gaging approach was reexamined, and an onsite visual observation program was investigated, but both were concluded to be technically and economically infeasible.

4 Evaluation and Recommendations

Monitoring Effort

Currents

The current monitoring satisfied the objective of determining the flushing characteristics of the breakwater, given the limited scope of the field effort. Volumetric flushing estimates have not been presented herein, since this study has shown that a more quantitative description of the flushing characteristics (such as that required for a circulation model study) would require greater density of data for the northern and southern ends of the marina, over a wider range of tide and wind conditions. The type of recording electromagnetic current meter used for this study would be excellent for a more comprehensive study if enough were deployed to cover the boundaries and internal points of interest.

Structural integrity

Monitoring efforts for structural integrity were adequate, given the constraints of the study. If side-scan surveys are performed in the future, an improvement would be to conduct them at extreme high tides to permit complete breakwater coverage and to lessen the risk of towfish damage. Future baffle opening surveys could be compared with the data presented herein to obtain quantitative scour information if a side-scan survey indicated development of scour problems. Given the small tide and wind-induced current velocities, scour development is unlikely, except possibly during a prolonged high-wave event (from standing wave-induced bottom velocities along the outside of the wall and through the openings). An improvement over spot checks of cap elevation would be to install brass disks in the cap and survey their three-dimensional position as part of a periodic inspection program, and after any major earthquake activity. Future visual inspections should pay careful attention to the hairline cracks in the cap, and should be documented photographically according to a retrievable position identification system.

Waves

The monitoring effort did not produce quantitative information on transmitted waves. In hindsight, the planning process relied too heavily on anecdotal information such as operator estimates of boat wake generation and the scope of the project did not allow enough flexibility or pre-testing of the procedure before the full-scale commitment of personnel and equipment. However, the experiment yielded some good qualitative information and the knowledge gained can be used to make the approach more workable for a future application where boat wakes are the primary concern. At this site, higher resolution pressure sensors or a different type of wave sensor (to measure water surface variation directly or even look at particle velocities instead) coupled with a more sophisticated data analysis and careful timing with respect to tides may be more successful.

Breakwater Performance and Design

Currents

Spud Point breakwater meets its performance criteria with respect to currents, since currents through the breakwater were measurable and exchange was clearly taking place. The reported pre-breakwater dominant pattern of north-to-south flow apparently has been preserved. A baffled design should be considered for lower energy environments where good circulation is critical to acceptability of the proposed structure.

Structural integrity

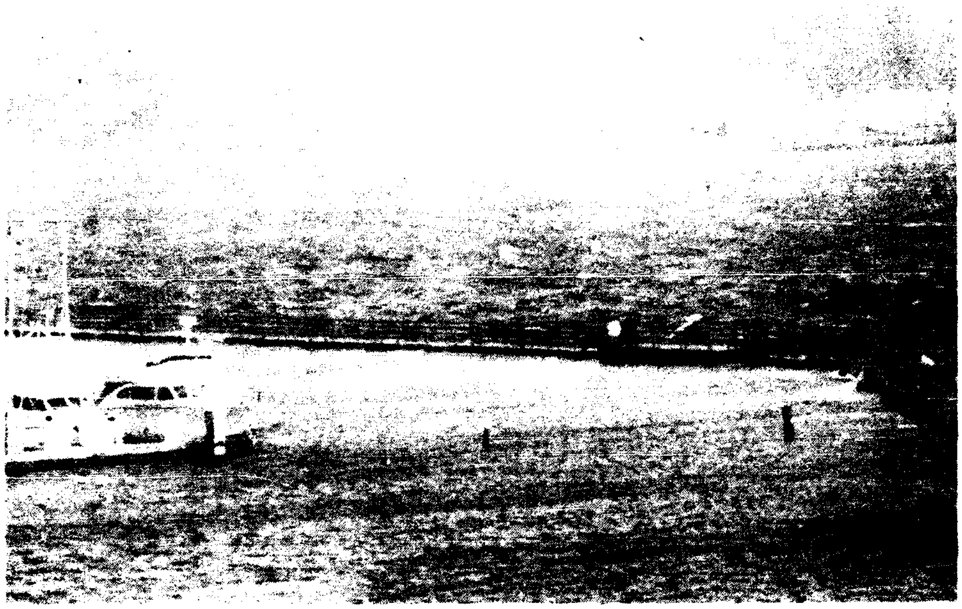
The breakwater has been successful in meeting the structural integrity performance criteria, assuming that the hairline cracking observed along the top of the cap at the bents is superficial and is not progressing or resulting in corrosion of the underlying reinforcement.

Waves

The breakwater appears to be meeting its performance criteria for wave transmission. Although this MCCP effort did not quantify the wave attenuation, the difficulty experienced in producing transmitted waves is evidence that the breakwater is providing significant wave attenuation. The marina operator reports that the breakwater has been very satisfactory, and that only the largest tenant boat, the 65-ft-long "Sea Angler," produces a wake large enough to penetrate the breakwater and cause rolling of the docked boats. Figure 19 (a - f) is convincing evidence of the breakwater's wind wave attenuation performance under severe conditions. Even with overtopping, the water inside the marina remained calm.

Although the transmitted wave heights were small during the experiment, the significant rolling of some of the boats due to the largest wakes suggests that parameters other than wave *height* may be of interest for wake or wave transmission criteria. It is unknown whether wind waves of similar height would have caused the rolling. The boats were docked so that they were broadside to the breakwater. One explanation may be that the baffled type of breakwater reduces vertical water particle motions and surface disturbances, yet allows enough horizontal motion to pass through the breakwater in the lower part of the water column to cause lateral motion in docked vessels. This emphasizes that for some protected breakwaters, protection against wakes may govern the design more than protection against wind waves.

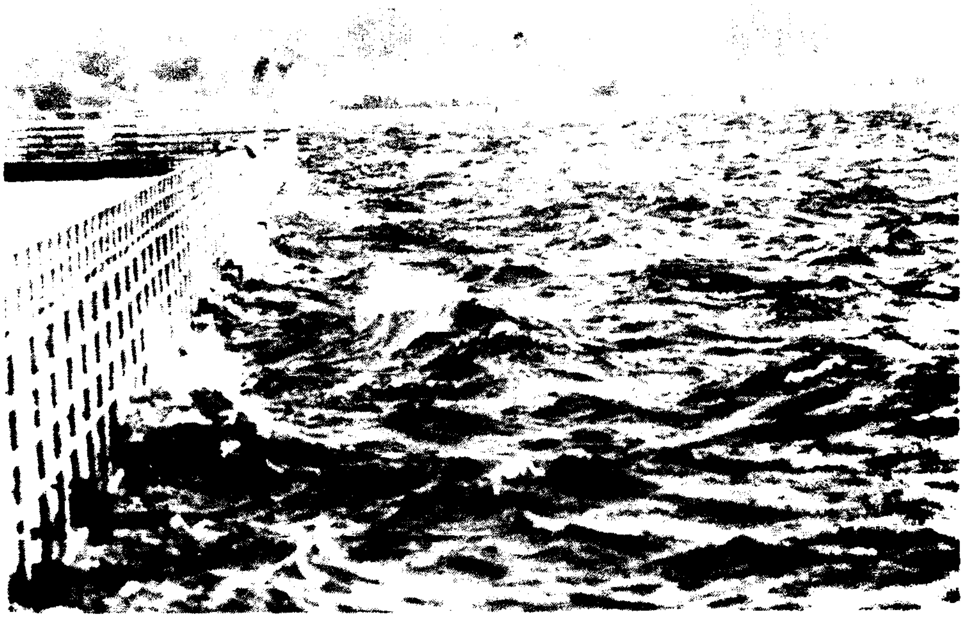
Based on the experimental, anecdotal, and photographic evidence, it seems that the breakwater is performing acceptably and Wiegel's nomograph (Wiegel 1960, 1964) was suitable for its design. Since the relative contribution of site-specific conditions to the successful performance of the Spud Point breakwater is unknown, the use of the nomograph and the baffled structure may not be more widely applicable except in preliminary studies. Physical model testing should be considered where costs or risks are high, until a more comprehensive evaluation of the theory and performance of baffled breakwaters has been completed.



a. View of seaward leg looking toward town of Bodega Bay

b. View of shoreward leg

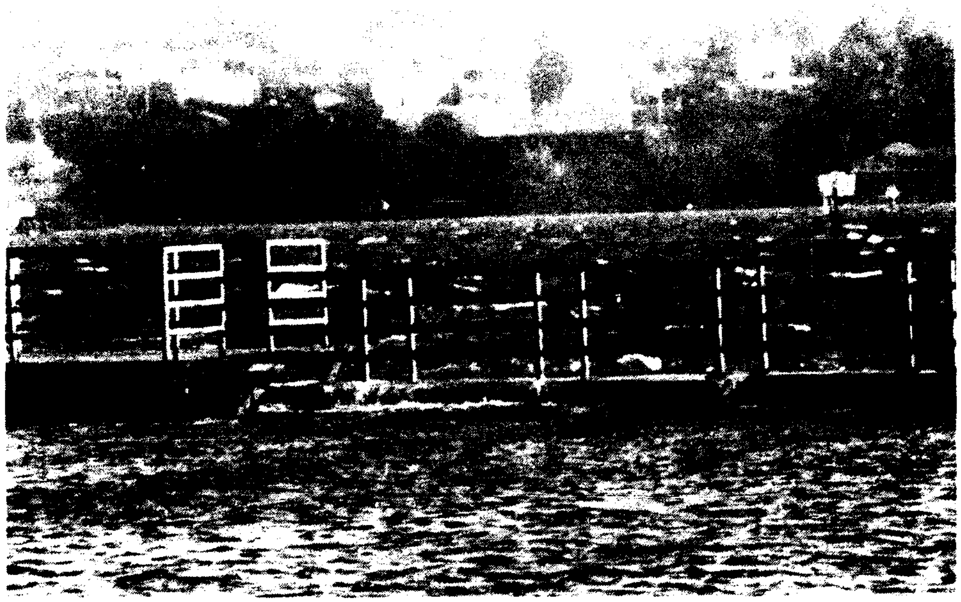
Figure 19. Breakwater in storm conditions at high tide (Sheet 1 of 3)



c. View of shoreward leg showing outside wave action

d. View of shoreward leg showing relatively calm inside conditions despite overtopping

Figure 19. (Sheet 2 of 3)



e. View of breakwater near bend point from inside marina

f. View of shoreward leg and walkway

Figure 19. (Sheet 3 of 3)

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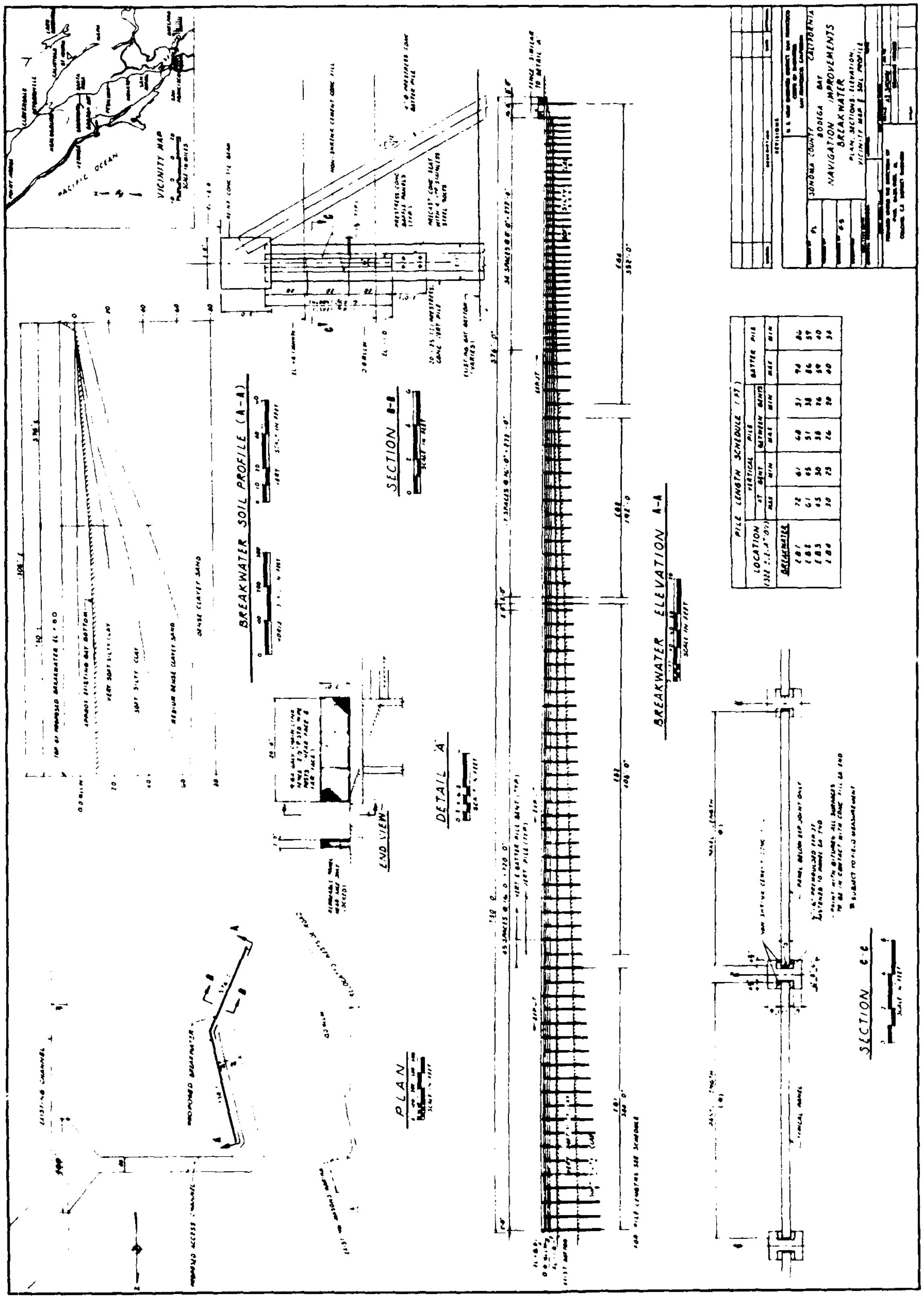
Appendix A
Breakwater Plan from General Design Memorandum (US Army Engineer District, San Francisco 1981)


VICINITY MAP
PACIFIC OCEAN
BREAKWATER SOIL PROFILE (A-A)
SECTION B-B
DETAIL 'A'
END VIEW
PLAN
BREAKWATER ELEVATION A-A
SECTION C-C
PILE LENGTH SCHEDULE
SONOMA COUNTY BODEGA BAY CALIFORNIA
NAVIGATION IMPROVEMENTS
BREAKWATER